POST-LOCAL BUCKLING OF STEEL PLATES IN CONCRETE-FILLED THIN-WALLED STEEL TUBULAR COLUMNS UNDER BIAXIAL LOADING

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ABSTRACT: This paper studies the post-local buckling behaviour of steel plates in concrete-filled thin-walled steel tubular columns under biaxial loading using the finite element method. Geometric and material nonlinear finite element analyses are undertaken to investigate the post-local buckling strengths of imperfect steel plates under stress gradients. Based on the results obtained from the nonlinear finite element analyses, a set of effective width formulas are proposed for determining the ultimate strengths of steel plates in concrete-filled steel tubular columns. The proposed effective width formulas are verified by comparisons with formulas developed by other researchers and can be used directly in the design of steel plates in concrete-filled steel tubular beam-columns.

KEYWORDS: composite column; effective width; post-local buckling; steel plates; ultimate strength.

1. INTRODUCTION

Thin steel plates in concrete-filled steel tubular (CFST) columns under biaxial loading are subjected to stress gradients. The local bucking behaviour of plates under eccentric loading has been studied by Walker [1]. Rhodes et al. [2] investigated the local and post-local buckling behaviour of plates with initial imperfections. Usami [3] proposed effective width formulas for predicting the ultimate strength of simply supported steel plates in compression and bending. Shanmugam et al. [4] investigated the ultimate loads of thin-walled steel box beam-columns including local buckling effects.

The local buckling behaviour of steel plates in CFST columns is characterized by the outward buckling mode. Ge and Usami [5] performed nonlinear finite element analyses on short thin-walled CFST columns and proposed ultimate strength formula for steel plates in uniform compression. Wright [6] derived the limiting width-to-thickness ratios for proportioning steel plates in contact with concrete. The ultimate load behaviour of CFST columns with local buckling effects has been studied experimentally by Uy and Bradford [7], Bridge et al. [8] and Uy [9]. Liang and Uy [10] and Liang et al. [11,12] investigated the local buckling behaviour of steel plates in composite members and proposed design formulas for determining the ultimate strengths of steel plates in such members.

Steel plates in concrete-filled thin-walled steel tubular columns under biaxial loading are subjected to stress gradients. This paper studies the post-local buckling strengths of steel plates under edge stress gradients in concrete-filled steel box beam-columns by using the geometric and material nonlinear finite element analyses. Based on the results obtained from the nonlinear finite element analyses, a set of effective width formulas are proposed for determining the ultimate strengths of steel plates in concrete-filled steel box beam-columns. The proposed design formulas are examined against available design formulas reported in the literature.

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2. FINITE ELEMENT ANALYSIS

2.1 GENERAL

The finite element code STRAND7 [13] was used to investigate the post-local buckling strengths of steel plates in thin-walled CFST beam-columns. The four edges of a web or flange in a CFST beam-column were assumed to be clamped due to the restraint provided by the concrete core [10]. The von Mises yield criterion was adopted in the nonlinear analysis to treat the material plasticity of steel plates. An eight-node quadrilateral plate/shell element was employed in all analyses. A 10 × 10 mesh was used in all analyses and was found to be adequate to yield accurate results for use in practice.

2.2 INITIAL IMPERFECTIONS

In the present study, the form of initial out-of-plane deflections was taken as the first local buckling mode. The maximum magnitude of the initial geometric imperfections at the plate centre was taken as $w_0 = 0.1t$ for steel plates in concrete-filled steel tubular beam-columns. An idealized residual stress pattern in a concrete-filled welded steel tubular column was adopted in the model in which the compressive residual stress was taken as 25 percent of the yield strength of the steel plate [10]. Residual stresses were incorporated in the finite element model by prestressing.

2.3 MATERIAL STRESS-STRAIN LAW FOR STEEL PLATES

A welded steel plate displays a rounded stress-strain form that differs from the tensile test behaviour of a coupon without residual stresses as reported by Liang and Uy [10]. In the present study, the rounded stress-strain curve of steel plates with residual stresses was modeled using the Ramberg-Osgood formula [14], which is expressed by

$$\varepsilon = \frac{\sigma}{E}\left[1 + \frac{3}{7}\left(\frac{\sigma}{\sigma_{0.7}}\right)^n\right] \tag{1}$$

where σ and ε are the uniaxial stress and strain respectively, E is the Young's modulus, $\sigma_{0.7}$ is the stress corresponding to $E_{0.7} = 0.7E$, and n is the knee factor that defines the sharpness of the knee in the stress-strain curve. The knee factor n = 25 was used in Equation (1) to account for the isotropic strain hardening of steel plates [10].

3. STEEL PLATES UNDER EDGE COMPRESSION

The post-local buckling behaviour of steel plates under gradient compressive stresses as shown in Figure 1 is studied in this section. The stress gradient coefficient α is defined as the ratio of the minimum edge stress (σ_2) to the maximum edge stress (σ_1). Stress gradient coefficients ranging from 0.0 to 0.2, 0.4, 0.6, 0.8 and 1.0 were considered. Square steel plates (500 × 500 mm) with initial geometric imperfections and welding residual stresses were studied. The thickness of the steel plates was varied to give different *b/t* ratios ranging from 30 to 100. The yield strength of steel plates was 300 MPa and the Young's modulus was 200 GPa.

Load-lateral deflection curves for steel plates with various *b/t* ratios under a stress gradient of $\alpha = 0.8$ are presented in Figure 2(a). The figure shows that all steel plates considered cannot attain the yield strength because of the effects of initial imperfections and stress gradients. The stiffness, critical local buckling strength and ultimate strength of steel plates generally decrease with an increase in the plate

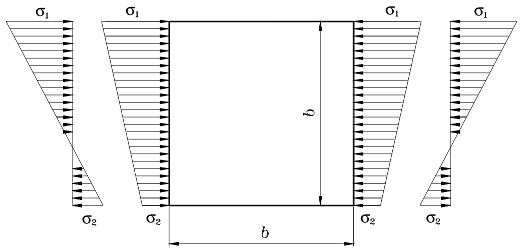
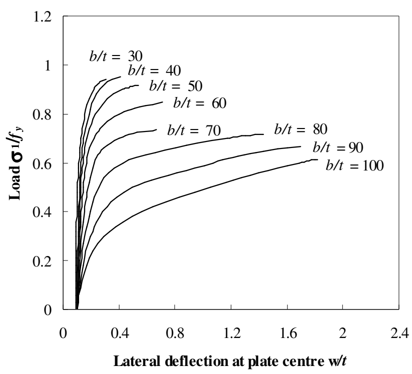
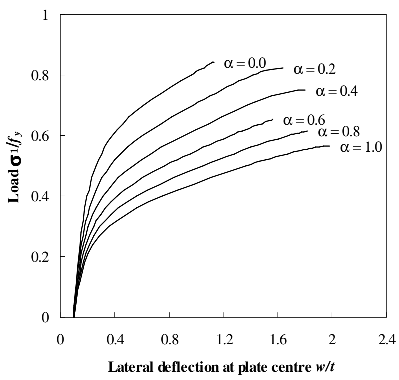


Figure 1. Clamped steel plates under edge compression and in-plane bending



(a)



(b)

Figure 2. Load-defection curves for steel plates under stress gradients: (a) $\alpha = 0.8$; (b) *b/t* =100.

width-to- thickness ratios. However, steel plates with the *b/t* ratios of 30 and 40 can attain the same ultimate stress, for these stocky plates undergo yielding only. It appears from Figure 2(a) that the ultimate strength of a steel plate with a *b/t* ratio of 100 is only 61.4 percent of its yield strength. Apparently, local buckling significantly reduces the ultimate strength of slender steel plates.

Fig. 2(b) demonstrates the effects of stress gradient coefficients on the load-deflection behaviour of steel plates with a *b/t* ratio of 100. The figure shows that the lateral stiffness of the steel plate is reduced when increasing the stress gradient coefficient α. It is also seen that increasing the stress gradient coefficient α reduces the ultimate strength of the steel plate. Figure 3 provides the ultimate strengths of steel plates subjected to edge compression in concrete-filled steel box beam-columns. It can be seen from Figure 3 that for steel plates under the same stress gradient the ultimate strength of steel plates decreases with an increase in the plate width-to-thickness ratio. As expected, increasing the stress gradient factor (α) would reduce the ultimate strength of a steel plate regardless of its width-to-thickness ratio.

4. STEEL PLATES UNDER IN-PLANE BENDING

The post-local buckling strengths of steel plates under in-plane bending as depicted in Figure 1 are investigated here. The geometry and material properties used in the analyses were the same as those presented in the preceding section. The stress gradient coefficient was varied from 0.2 to 0.4, 0.6, 0.8 and 1.0. The effect of stress gradient coefficients on the load-deflection behaviour of the steel plate with a *b/t* ratio of 80 is schematically demonstrated in Figure 4. It is seen that reducing the stress gradient coefficient increases the lateral deflection of the plate under the same loading level.

5. DESIGN FORMULAS FOR ULTIMATE STRENGTH

It can be seen from Figure 3 that the ultimate strength of a steel plate with prescribed geometric

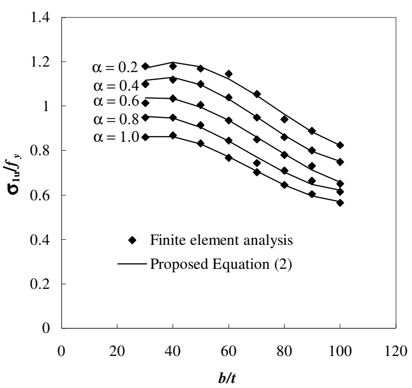


Figure 3. Ultimate strengths of steel plates

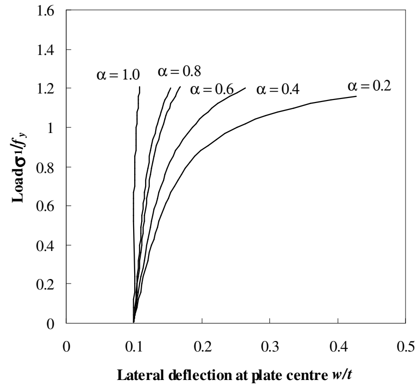


Figure 4. Steel plates under in-plane bending

imperfections and residual stresses is a function of the *b/t* ratio, stress gradient coefficient (α) and the yield strength (f_y). A single formula is proposed to approximately express the ultimate strength of steel plates with stress gradient coefficients greater than zero as follows:

$$\frac{\sigma_{1u}}{f_y} = (1 + 0.5\phi)\frac{\sigma_u}{f_y} \qquad (0 \le \phi < 1.0) \tag{2}$$

where $\phi = 1 - \alpha$ and σ_u is the ultimate stress of steel plates under uniform compression, which can be determined by

$$\frac{\sigma_u}{f_y} = 0.5554 + 0.02038\left(\frac{b}{t}\right) - 3.944\times10^{-4}\left(\frac{b}{t}\right)^2 + 1.921\times10^{-6}\left(\frac{b}{t}\right)^3 \tag{3}$$

Figure 3 shows that the ultimate strengths of steel plates predicted by the proposed Equations (2) and (3) are compared well with those obtained from the finite element analysis. Figure 5 schematically depicts the effective width of a thin steel plate in the post-local buckling regime under compression and in-plane bending. Based on the results obtained from the nonlinear finite element analyses and the proposed ultimate strength formulas, effective width formulas for determining the ultimate strength of clamped steel plates under edge compressive stress gradients in concrete-filled steel tubular beam-columns are proposed as

$$\frac{b_{e1}}{b} = 0.2777 + 0.01019\left(\frac{b}{t}\right) - 1.972\times10^{-4}\left(\frac{b}{t}\right)^2 + 9.605\times10^{-7}\left(\frac{b}{t}\right)^3 \quad \text{for } \alpha > 0.0 \tag{4a}$$

$$\frac{b_{e1}}{b} = 0.4186 - 0.002047\left(\frac{b}{t}\right) + 5.355\times10^{-5}\left(\frac{b}{t}\right)^2 - 4.685\times10^{-7}\left(\frac{b}{t}\right)^3 \quad \text{for } \alpha = 0.0 \tag{4b}$$

$$\frac{b_{e2}}{b} = (1 + \phi)\frac{b_{e1}}{b} \tag{5}$$

where b_{e1} and b_{e2} are the effective widths as depicted in Figure 5. Note that for $(b_{e1} + b_{e2}) \ge b$, the steel plate is fully effective in carrying loads and the ultimate strength of the steel plate can be determined using Equations (2) and (3).

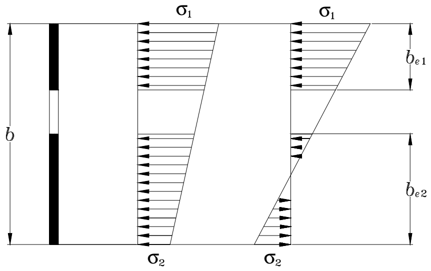
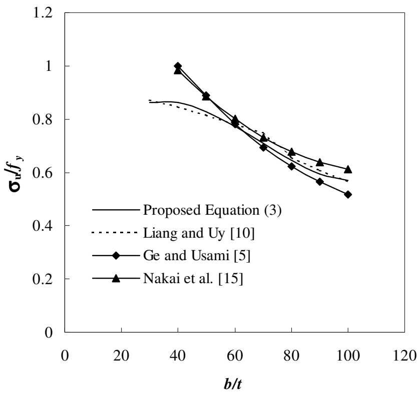
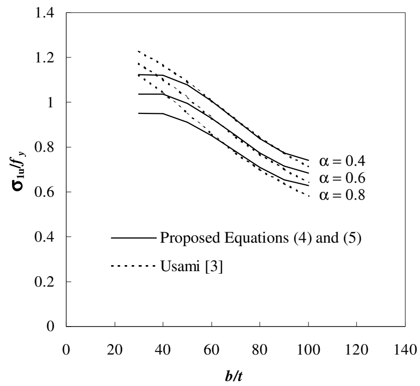


Figure 5. Effective width of steel plate under compression and in-plane bending

6. COMPARISONS WITH EXISTING FORMULAS

The proposed design Equation (3) are compared with those given by Liang and Uy [10] and Ge and Usami [5] and Nakai et al. [15] for steel plates under uniform compression. It can be seen from Figure 6(a) that the proposed design formula compares very well with those of Liang and Uy [10]. For steel plates with a *b/t* ratio greater than 60, the ultimate strength of the plates predicted by Equation (3) is between those calculated using equations by Usami [5] and Nakai et al [15]. The proposed design formula yields conservative predictions of the ultimate strengths for steel plates with a *b/t* ratio less than 60.





(a) (b)

Figure 6. Comparison of the proposed formulas with those presented by other researchers: (a) uniform compression; (b) Non-uniform compression

The proposed effective width formulas for clamped steel plates under non-uniform compression are compared with formulas developed by Usami [3]. The width-to-thickness ratio parameter β_n in Usami's equations was calculated using the elastic local buckling coefficient k_n for clamped steel plates under non-uniform compression, which is proposed based on the finite element analysis as

$$k_n = 18.89 - 14.38\alpha + 5.3\alpha^2 \tag{6}$$

Figure 6(b) provides a comparison of the ultimate strengths predicted by the proposed effective width formulas Equations (4) and (5) and those given by Usami [3] with buckling coefficient k_n determined by Equation (6) for clamped plates with stress gradient coefficients of 0.4, 0.6 and 0.8. The initial geometric imperfection was taken as $w_0 = 0.1t$ and the compressive residual stress was assumed to be $0.25f_y$ for steel plates (500 × 500 mm). Figure 6(b) shows that the proposed effective width formulas compare very well with those given by Usami [3].

7. CONCLUSIONS

The post-local buckling behaviour of steel plates in concrete-filled thin-walled steel tubular columns under biaxial loads has been investigated by undertaking the geometric and material nonlinear finite element analyses in this paper. The effects of stress gradient coefficients and width-to-thickness ratios on the ultimate strengths of steel plates in concrete-filled steel box columns were investigated. Based on the results obtained from the nonlinear finite element analyses, a set of design formulas were proposed for determining the ultimate strengths of steel plates under compression and in-plane bending. These design formulas can be used directly in the design of steel plates in concrete-filled thin-walled steel tubular beam-columns and are suitable for inclusion in composite design codes. Moreover, these effective width formulas can be incorporated in advanced analysis methods to account for local buckling effects on concrete-filled steel tubular columns and composite frames.

8. REFERENCES

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